POS. 9: SK'15, LENER, BSP.3A, S.263

detailed problems acc. to Eurocode 3

EC 3-1-5 (12.10), NA: Deutschland

steel grade

steel grade S 355

cross-section

beam: parameter (I-section):

h = 3080.0 mm, t_w = 15.0 mm, b_f = 800.0 mm, t_f = 40.0 mm

longitudinal stiffeners: number n_{st} = 1

section parameters (flat steel):

h = 250.0 mm, t = 25.0 mm

distance of stiffener to the top edge of beam $d_{st,0}$ = 2540.0 mm

parameters

length of buckling field a = 300.0 cm

method of effective cross-sectional area

rigid support stiffener

verification at intermediate support

calculation of buckling factors acc. to EC 3-1-5

effective cross-sectional properties: A_{eff} solely from compression, W_{eff} solely from bending

verification of stability acc. to EC 3-1-1, 6.3

loading

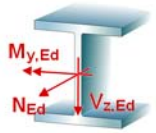
internal forces and moments refering to the stiffened cross-section:

Lk 1: plated structures- + shear buckling (chap. 5.1.1-4)

N_{Ed} = -4000.0 kN M_{Ed} = -33760.0 kNm V_{Ed} = 3437.5 kN

Lk 2: M-N-V-interaction (chap. 5.1.5)

N_{Ed} = -4000.0 kN M_{Ed} = -32150.0 kNm V_{Ed} = 3288.0 kN



partial safety factors for material

resistance of cross-sections γ_{M0} = 1.00

resistance of members in stability failure γ_{M1} = 1.10

verifications of buckling resistance

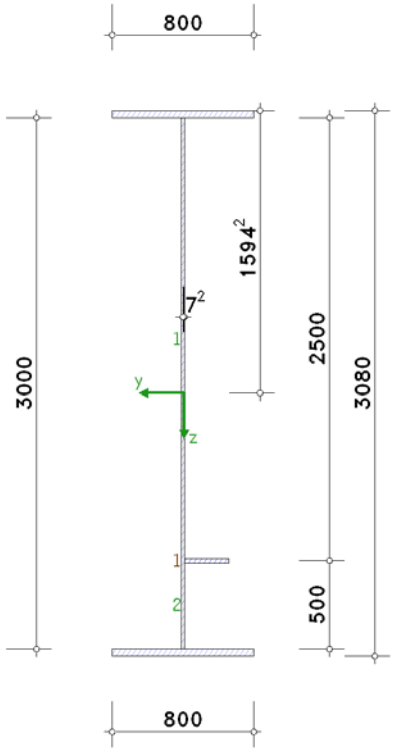
assumption: flange induced web buckling is excluded.

assumption: local buckling of stiffeners is excluded.

assumption: rotational ability of stiffeners for stress redistribution is sufficient.

assumption: plate area is supported rigidly.

the assumptions are not checked !!



method of effective cross-sectional area

EC 3-convention, compressive stresses positive

shear distortions are ignored.

cross-sectional properties: A = 1152.50 cm², z_s = 1594.2 mm, I_y = 18753552.18 cm⁴, y_s = 7.2 mm, I_z = 355050.53 cm⁴

maximum/minimum stresses: σ_o = -252.3 N/mm², σ_u = 302.2 N/mm², τ = 76.4 N/mm²

section class: 4 ⇒ verification of plate buckling required !!

plate buckling

effective cross-sectional area for N_{Ed} = 4000.0 kN, M_{Ed} = 0

flange top:

section class 3 for 8.14 < c/t = 9.81 < 11.20

effective width $b_{c,eff}$ = b = 392.5 mm

flange bottom:

section class 3 for 8.14 < c/t = 9.81 < 11.20

effective width $b_{c,eff}$ = b = 392.5 mm

web:

single buckling field 1:

section class 4 for 34.17 < c/t = 165.83

critical buckling stress $\sigma_{cr,p} = k_\sigma \cdot \sigma_E$ = 27.6 N/mm², σ_E = 6.9 N/mm², k_σ = 4.00

buckling slenderness ratio $\lambda_p = (f_y/\sigma_{cr,p})^{1/2}$ = 3.586

reduction factor $\rho = (\lambda_p - 0.055\cdot(3+\psi))/\lambda_p^2$ = 0.262 ≤ 1 for $\lambda_p > 0.5 + (0.085-0.055\cdot\psi)^{1/2}$ = 0.673, ψ = 1.000

effective width $b_{c,eff} = \rho \cdot b$ = 651.1 mm (b_{e1} = 325.6 mm, b_{e2} = 325.6 mm)

single buckling field 2:

section class 3 for 30.92 < c/t = 32.50 < 34.17

effective width $b_{c,eff}$ = b = 487.5 mm (b_{e1} = 243.8 mm, b_{e2} = 243.8 mm)

overall buckling field, stiffener 1:

EC 3-1-5, appendix A.2.2 (fictitious member with elastic bedding):

critical buckling stress $\sigma_{cr,p} = \sigma_{cr,p,sl}\cdot\sigma_1/\sigma_{sl}$ = 958.8 N/mm², σ_1/σ_{sl} = 1.000, $\sigma_{cr,p,sl}$ = 958.8 N/mm²

buckling slenderness ratio $\lambda_p = (\beta_A \cdot f_y/\sigma_{cr,p})^{1/2}$ = 0.440, $\beta_A = A_{sl,eff}/A_{sl}$ = 0.524

reduction factor ρ = 1 for $\lambda_p < 0.5 + (0.085-0.055\cdot\psi)^{1/2}$ = 0.673, ψ = 1.000

critical buckling stress $\sigma_{cr,c} = \sigma_{cr,c,sl}\cdot\sigma_1/\sigma_{sl}$ = 947.1 N/mm², σ_1/σ_{sl} = 1.000, $\sigma_{cr,c,sl}$ = 947.1 N/mm²

buckling slenderness ratio $\lambda_c = (\beta_A \cdot f_y/\sigma_{cr,c})^{1/2}$ = 0.443, $\beta_A = A_{sl,eff}/A_{sl}$ = 0.524

reduction factor χ_c = 0.844 ≤ 1 for $\lambda_c > 0.2$

final reduction factor $\rho = (\rho-\chi_c)\cdot\xi\cdot(2-\xi) + \chi_c$ = 0.848 with ξ = 0.012

effective widths of adjacent buckling fields $b_{1,e2,eff} = \rho \cdot b_{1,c,eff}$ = 275.9 mm, $b_{2,e1,eff} = \rho \cdot b_{2,c,eff}$ = 206.6 mm

effective area of stiffener $A_{sl,eff} = \rho \cdot A_{st}$ = 52.97 cm²

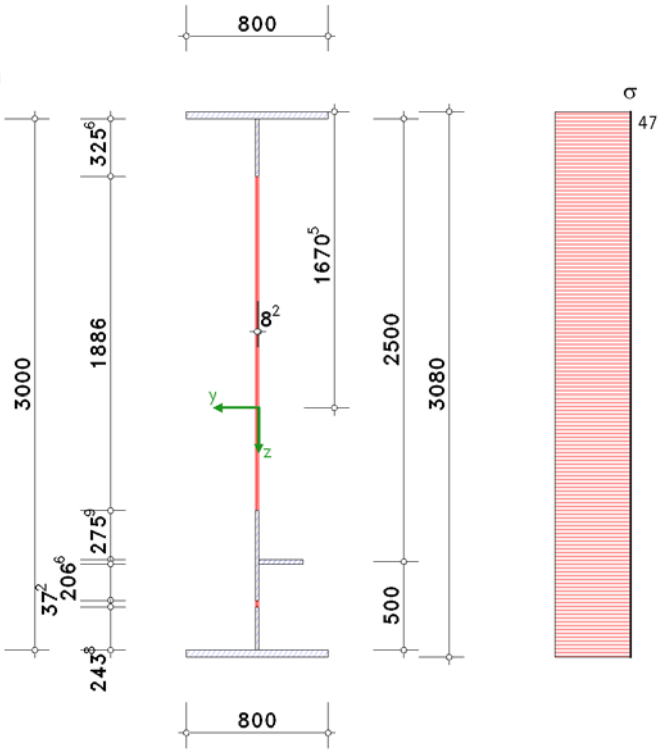
limit loads refering to the reduced cross-section:

distance of centroid from top $z_{s,eff}$ = 1670.5 mm

differential moment by Δz_s = 76.3 mm: ΔM_{Ed} = 305.22 kNm

cross-sectional area A_{eff} = 854.49 cm²

load capacities $N_{Rd} = (f_y \cdot A_{eff}) / \gamma_{M1}$ = 27576.68 kN



effective section modulus for M_{Ed} = 33760.0 kNm, N_{Ed} = 0

flange top:

effective width $b_{t,eff}$ = b = 392.5 mm

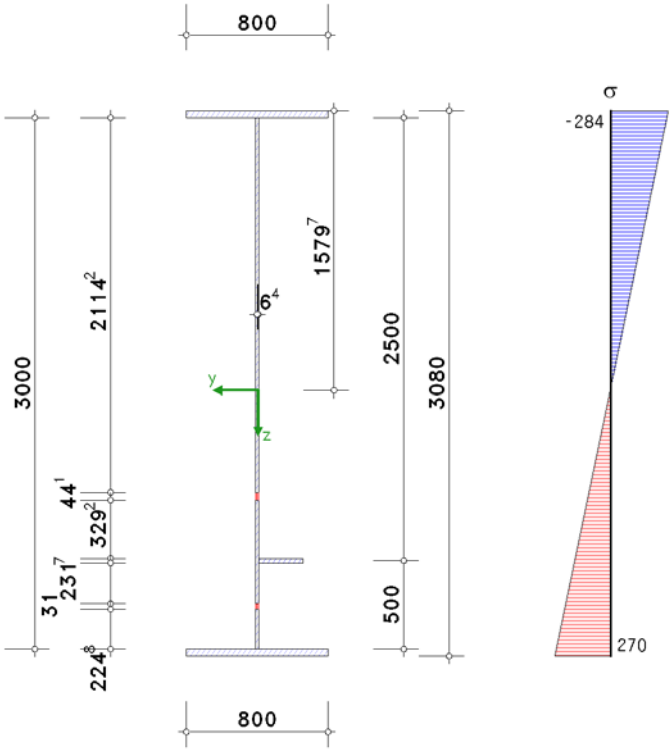
flange bottom:

section class 3 for 8.14 < c/t = 9.81 < 11.20

effective width $b_{c,eff}$ = b = 392.5 mm

web:
single buckling field 1:
section class 3 for α = 0.375 and 90.00 < c/t = 165.83 < 173.51
effective width $b_{c,eff} = (\rho \cdot b) / (1-\psi)$ = 933.3 mm (b_{e1} = 373.3 mm, b_{e2} = 560.0 mm), $b_{t,eff}$ = 1554.2 mm, ψ = -1.665
single buckling field 2:
section class 3 for 30.92 < c/t = 32.50 < 38.50
effective width $b_{c,eff}$ = b = 487.5 mm (b_{e1} = 224.8 mm, b_{e2} = 262.7 mm)
overall buckling field, stiffener 1:
EC 3-1-5, appendix A.2.2 (fictitious member with elastic bedding):
critical buckling stress $\sigma_{cr,p} = \sigma_{cr,p,sl} \cdot \sigma_1/\sigma_{sl}$ = 2210.9 N/mm², σ_1/σ_{sl} = 1.529, $\sigma_{cr,p,sl}$ = 1446.3 N/mm²
buckling slenderness ratio $\lambda_p = (\beta_A \cdot f_y/\sigma_{cr,p})^{1/2}$ = 0.401, $\beta_A = A_{sl,eff}/A_{sl}$ = 1.000
reduction factor ρ = 1 for $\lambda_p < 0.5 + (0.085-0.055 \cdot \psi)^{1/2}$ = 0.880, ψ = -1.075
critical buckling stress $\sigma_{cr,c} = \sigma_{cr,c,sl} \cdot \sigma_1/\sigma_{sl}$ = 2178.6 N/mm², σ_1/σ_{sl} = 1.529, $\sigma_{cr,c,sl}$ = 1425.2 N/mm²
buckling slenderness ratio $\lambda_c = (\beta_A \cdot f_y/\sigma_{cr,c})^{1/2}$ = 0.404, $\beta_A = A_{sl,eff}/A_{sl}$ = 1.000
reduction factor χ_c = 0.878 ≤ 1 for λ_c > 0.2
final reduction factor $\rho = (\rho-\chi_c) \cdot \xi \cdot (2-\xi) + \chi_c$ = 0.882 with ξ = 0.015
effective widths of adjacent buckling fields $b_{1,e2,eff} = \rho \cdot b_{1,c,eff}$ = 329.2 mm, $b_{2,e1,eff} = \rho \cdot b_{2,c,eff}$ = 231.7 mm
effective area of stiffener $A_{sl,eff} = \rho \cdot A_{st}$ = 55.12 cm²

limit loads refering to the reduced cross-section:
distance of centroid from top $z_{s,eff}$ = 1579.7 mm
second moment of area $I_{y,eff}$ = 18595061.34 cm⁴
section modulus $W_{y,eff}$ = 125619.21 cm³
load capacities $M_{Rd} = (f_y \cdot W_{eff}) / \gamma_{M1}$ = 40540.74 kNm



verification (M-N-interaction)
acting moment $M_{Ed}' = M_{Ed} - \Delta M_{Ed}$ = 33454.78 kNm
$|N_{Ed}|/N_{Rd} + |M_{Ed}'|/M_{Rd,u}$ = 0.145 + 0.825 = 0.970 < 1 **ok.**

shear buckling
single buckling field 1:
shear buckling: h_w/t_w = 166.67 > $72 \cdot \varepsilon/\eta$ = 48.82 ⇒ particular verification is required !!
buckling factor of shear for a/h_w = 1.20 > 1: $k_\tau = 5.34 + 4/(a/h_w)^2$ = 8.12
critical buckling stress of shear $\tau_{cr,p} = k_\tau \cdot \sigma_E$ = 55.5 N/mm², σ_E = 6.8 N/mm²
modified slenderness $\lambda_w = 0.76 \cdot (f_y/\tau_{cr,p})$ = 1.923
reduction factor $\chi_w = 1.37/(0.7+\lambda_w)$ = 0.522 for $\lambda_w \geq 1.08$
resistance $V_{bw,Rd} = (\chi_w \cdot f_y \cdot h_w \cdot t_w) / (3^{1/2} \cdot \gamma_{M1})$ = 4379.87 kN
design value of resistance $V_{b,Rd} = V_{bw,Rd}$ = 4379.9 kN
single buckling field 2:
shear buckling: h_w/t_w = 33.33 ≤ $72 \cdot \varepsilon/\eta$ = 48.82 **ok.**
overall buckling field:
shear buckling: h_w/t_w = 200.00 > $31 \cdot \varepsilon \cdot k_\tau^{1/2}/\eta$ = 85.14, k_τ = 16.41 ⇒ particular verification is required !!
contribution of the flanges:
resisting moment $M_{f,Rd} = f_{N,f} \cdot M_{f,k}/\gamma_{M0}$ = 28454.4 kNm, $M_{f,k}$ = 34534.4 kNm, $f_{N,f}$ = 0.82
$M_{Ed} \geq M_{f,Rd}$: flanges fully exploited, resistance $V_{bf,Rd}$ = 0
contribution of the web:
buckling factor of shear for a/h_w = 1.00 < 3: $k_\tau = 4.1 + (6.3+0.18 \cdot I_{sl}/(h_w \cdot t_w^3))/(a/h_w)^2 + 2.2 \cdot (I_{sl}/(h_w \cdot t_w^3))^{1/3}$ = 16.41
critical buckling stress of shear $\tau_{cr,p} = k_\tau \cdot \sigma_E$ = 77.9 N/mm², σ_E = 4.7 N/mm²
modified slenderness $\lambda_w = 0.76 \cdot (f_y/\tau_{cr,p})$ = 1.623
reduction factor $\chi_w = 1.37/(0.7+\lambda_w)$ = 0.590 for $\lambda_w \geq 1.08$

resistance $V_{bw,Rd} = (\chi_w \cdot f_y \cdot h_w \cdot t_w) / (3^{1/2} \cdot \gamma_{M1}) = 4945.45$ kN
design value of resistance $V_{b,Rd} = V_{bw,Rd} = 4945.4$ kN
load capacities $V_{b,Rd,min} = 4379.9$ kN, $V_{bw,Rd,min} = 4379.9$ kN
verification: $V_{Ed}/V_{b,Rd,min} = 0.785 < 1$ ok.

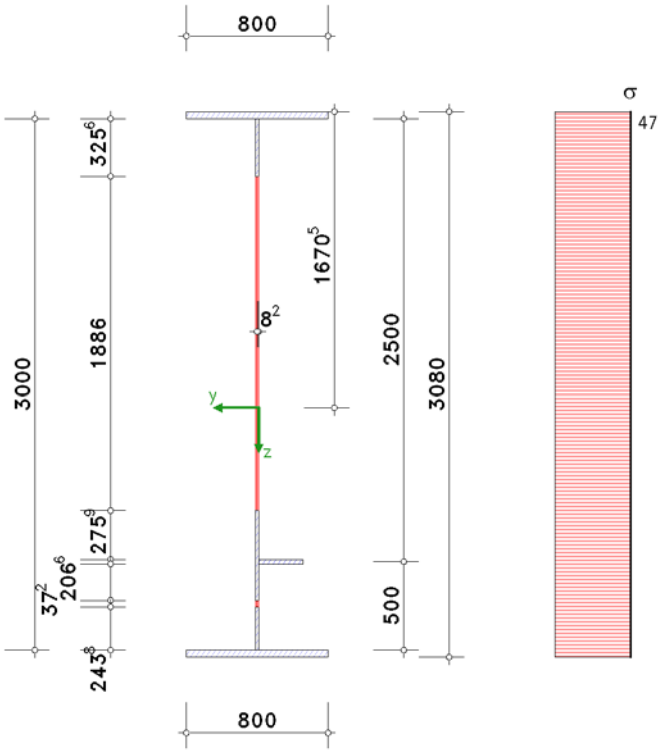
interaction between shear, internal moment and axial force
utilization due to shear buckling of the web $\eta_3 = V_{Ed}/V_{bw,Rd,min} = 0.785 > 0.5$
utilization due to plate buckling $\eta_1 = M_{Ed}/M_{pl,Rd} = 0.696 > \eta_{1,lim} = M_{f,Rd}/M_{pl,Rd} = 0.587$
with $M_{pl,Rd} = 48493.1$ kNm, $M_{f,Rd} = 28454.4$ kNm
verification: $\eta_1 + (1-\eta_{1,lim}) \cdot (2 \cdot \eta_3 - 1)^2 = 0.830 < 1$ ok.

total utilization: $U = 0.970 < 1$ ok.

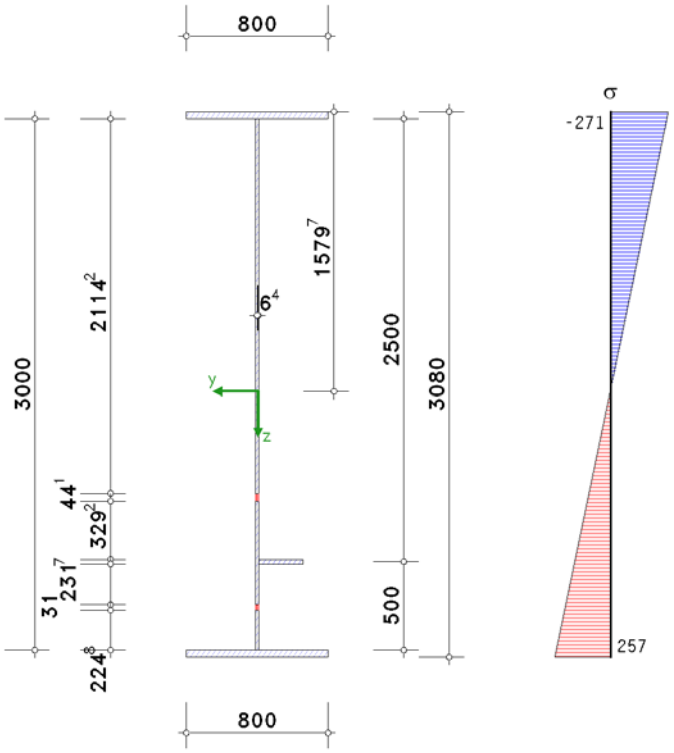
Lk 2: M-N-V-interaction (chap. 5.1.5)

method of effective cross-sectional area
EC 3-convention, compressive stresses positive
shear distortions are ignored.
cross-sectional properties: $A = 1152.50$ cm², $z_s = 1594.2$ mm, $I_y = 18753552.18$ cm⁴, $y_s = 7.2$ mm, $I_z = 355050.53$ cm⁴
maximum/minimum stresses: $\sigma_o = -238.6$ N/mm², $\sigma_u = 289.4$ N/mm², $\tau = 73.1$ N/mm²
section class: 4 ⇒ verification of plate buckling required !!

plate buckling
effective cross-sectional area for $N_{Ed} = 4000.0$ kN, $M_{Ed} = 0$
flange top:
section class 3 for $8.14 < c/t = 9.81 < 11.20$
effective width $b_{c,eff} = b = 392.5$ mm
flange bottom:
section class 3 for $8.14 < c/t = 9.81 < 11.20$
effective width $b_{c,eff} = b = 392.5$ mm
web:
single buckling field 1:
section class 4 for $34.17 < c/t = 165.83$
critical buckling stress $\sigma_{cr,p} = k_\sigma \cdot \sigma_E = 27.6$ N/mm², $\sigma_E = 6.9$ N/mm², $k_\sigma = 4.00$
buckling slenderness ratio $\lambda_p = (f_y/\sigma_{cr,p})^{1/2} = 3.586$
reduction factor $\rho = (\lambda_p - 0.055 \cdot (3+\psi))/\lambda_p^2 = 0.262 \le 1$ for $\lambda_p > 0.5 + (0.085 - 0.055 \cdot \psi)^{1/2} = 0.673$, $\psi = 1.000$
effective width $b_{c,eff} = \rho \cdot b = 651.1$ mm ($b_{e1} = 325.6$ mm, $b_{e2} = 325.6$ mm)
single buckling field 2:
section class 3 for $30.92 < c/t = 32.50 < 34.17$
effective width $b_{c,eff} = b = 487.5$ mm ($b_{e1} = 243.8$ mm, $b_{e2} = 243.8$ mm)
overall buckling field, stiffener 1:
EC 3-1-5, appendix A.2.2 (fictitious member with elastic bedding):
critical buckling stress $\sigma_{cr,p} = \sigma_{cr,p,sl} \cdot \sigma_1/\sigma_{sl} = 958.8$ N/mm², $\sigma_1/\sigma_{sl} = 1.000$, $\sigma_{cr,p,sl} = 958.8$ N/mm²
buckling slenderness ratio $\lambda_p = (\beta_A \cdot f_y/\sigma_{cr,p})^{1/2} = 0.440$, $\beta_A = A_{sl,eff}/A_{sl} = 0.524$
reduction factor $\rho = 1$ for $\lambda_p < 0.5 + (0.085 - 0.055 \cdot \psi)^{1/2} = 0.673$, $\psi = 1.000$
critical buckling stress $\sigma_{cr,c} = \sigma_{cr,c,sl} \cdot \sigma_1/\sigma_{sl} = 947.1$ N/mm², $\sigma_1/\sigma_{sl} = 1.000$, $\sigma_{cr,c,sl} = 947.1$ N/mm²
buckling slenderness ratio $\lambda_c = (\beta_A \cdot f_y/\sigma_{cr,c})^{1/2} = 0.443$, $\beta_A = A_{sl,eff}/A_{sl} = 0.524$
reduction factor $\chi_c = 0.844 \le 1$ for $\lambda_c > 0.2$
final reduction factor $\rho = (\rho - \chi_c) \cdot \xi \cdot (2 - \xi) + \chi_c = 0.848$ with $\xi = 0.012$
effective widths of adjacent buckling fields $b_{1,e2,eff} = \rho \cdot b_{1,c,eff} = 275.9$ mm, $b_{2,e1,eff} = \rho \cdot b_{2,c,eff} = 206.6$ mm
effective area of stiffener $A_{sl,eff} = \rho \cdot A_{st} = 52.97$ cm²
limit loads refering to the reduced cross-section:
distance of centroid from top $z_{s,eff} = 1670.5$ mm
differential moment by $\Delta z_s = 76.3$ mm: $\Delta M_{Ed} = 305.22$ kNm
cross-sectional area $A_{eff} = 854.49$ cm²
load capacities $N_{Rd} = (f_y \cdot A_{eff}) / \gamma_{M1} = 27576.68$ kN



effective section modulus for M_{Ed} = 32150.0 kNm, N_{Ed} = 0
flange top:
  effective width $b_{t,eff}$ = b = 392.5 mm
flange bottom:
  section class 3 for 8.14 < c/t = 9.81 < 11.20
  effective width $b_{c,eff}$ = b = 392.5 mm
web:
  single buckling field 1:
    section class 3 for α = 0.375 and 90.00 < c/t = 165.83 < 173.51
    effective width $b_{c,eff} = (\rho \cdot b) / (1-\psi)$ = 933.3 mm (b_{e1} = 373.3 mm, b_{e2} = 560.0 mm), $b_{t,eff}$ = 1554.2 mm, ψ = -1.665
  single buckling field 2:
    section class 3 for 30.92 < c/t = 32.50 < 38.50
    effective width $b_{c,eff}$ = b = 487.5 mm (b_{e1} = 224.8 mm, b_{e2} = 262.7 mm)
  overall buckling field, stiffener 1:
    EC 3-1-5, appendix A.2.2 (fictitious member with elastic bedding):
    critical buckling stress $\sigma_{cr,p} = \sigma_{cr,p,sl} \cdot \sigma_1/\sigma_{sl}$ = 2210.9 N/mm², σ_1/σ_{sl} = 1.529, $\sigma_{cr,p,sl}$ = 1446.3 N/mm²
    buckling slenderness ratio $\lambda_p = (\beta_A \cdot f_y/\sigma_{cr,p})^{1/2}$ = 0.401, $\beta_A = A_{sl,eff}/A_{sl}$ = 1.000
    reduction factor ρ = 1 for $\lambda_p < 0.5 + (0.085-0.055 \cdot \psi)^{1/2}$ = 0.880, ψ = -1.075
    critical buckling stress $\sigma_{cr,c} = \sigma_{cr,c,sl} \cdot \sigma_1/\sigma_{sl}$ = 2178.6 N/mm², σ_1/σ_{sl} = 1.529, $\sigma_{cr,c,sl}$ = 1425.2 N/mm²
    buckling slenderness ratio $\lambda_c = (\beta_A \cdot f_y/\sigma_{cr,c})^{1/2}$ = 0.404, $\beta_A = A_{sl,eff}/A_{sl}$ = 1.000
    reduction factor χ_c = 0.878 ≤ 1 for λ_c > 0.2
    final reduction factor $\rho = (\rho-\chi_c) \cdot \xi \cdot (2-\xi) + \chi_c$ = 0.882 with ξ = 0.015
    effective widths of adjacent buckling fields $b_{1,e2,eff} = \rho \cdot b_{1,c,eff}$ = 329.2 mm, $b_{2,e1,eff} = \rho \cdot b_{2,c,eff}$ = 231.7 mm
    effective area of stiffener $A_{sl,eff} = \rho \cdot A_{st}$ = 55.12 cm²
limit loads refering to the reduced cross-section:
  distance of centroid from top $z_{s,eff}$ = 1579.7 mm
  second moment of area $I_{y,eff}$ = 18595061.34 cm⁴
  section modulus $W_{y,eff}$ = 125619.21 cm³
  load capacities $M_{Rd} = (f_y \cdot W_{eff}) / \gamma_{M1}$ = 40540.74 kNm



verification (M-N-interaction)
acting moment $M_{Ed}' = M_{Ed} - \Delta M_{Ed} = 31844.78$ kNm
$|N_{Ed}|/N_{Rd} + |M_{Ed}'|/M_{Rd,u} = 0.145 + 0.786 = 0.931 < 1$ **ok.**

shear buckling
single buckling field 1:
shear buckling: $h_w/t_w = 166.67 > 72\cdot\varepsilon/\eta = 48.82 \Rightarrow$ particular verification is required !!
buckling factor of shear for $a/h_w = 1.20 > 1$: $k_\tau = 5.34 + 4/(a/h_w)^2 = 8.12$
critical buckling stress of shear $\tau_{cr,p} = k_\tau\cdot\sigma_E = 55.5$ N/mm², $\sigma_E = 6.8$ N/mm²
modified slenderness $\lambda_w = 0.76\cdot(f_y/\tau_{cr,p}) = 1.923$
reduction factor $\chi_w = 1.37/(0.7+\lambda_w) = 0.522$ for $\lambda_w \geq 1.08$
resistance $V_{bw,Rd} = (\chi_w\cdot f_y\cdot h_w\cdot t_w) / (3^{1/2}\cdot\gamma_{M1}) = 4379.87$ kN
design value of resistance $V_{b,Rd} = V_{bw,Rd} = 4379.9$ kN
single buckling field 2:
shear buckling: $h_w/t_w = 33.33 \leq 72\cdot\varepsilon/\eta = 48.82$ **ok.**
overall buckling field:
shear buckling: $h_w/t_w = 200.00 > 31\cdot\varepsilon\cdot k_\tau^{1/2}/\eta = 85.14$, $k_\tau = 16.41 \Rightarrow$ particular verification is required !!
contribution of the flanges:
resisting moment $M_{f,Rd} = f_{N,f}\cdot M_{f,k}/\gamma_{M0} = 28454.4$ kNm, $M_{f,k} = 34534.4$ kNm, $f_{N,f} = 0.82$
$M_{Ed} \geq M_{f,Rd}$: flanges fully exploited, resistance $V_{bf,Rd} = 0$
contribution of the web:
buckling factor of shear for $a/h_w = 1.00 < 3$: $k_\tau = 4.1 + (6.3+0.18\cdot I_{sl}/(h_w\cdot t_w^3))/(a/h_w)^2 + 2.2\cdot(I_{sl}/(h_w\cdot t_w^3))^{1/3} = 16.41$
critical buckling stress of shear $\tau_{cr,p} = k_\tau\cdot\sigma_E = 77.9$ N/mm², $\sigma_E = 4.7$ N/mm²
modified slenderness $\lambda_w = 0.76\cdot(f_y/\tau_{cr,p}) = 1.623$
reduction factor $\chi_w = 1.37/(0.7+\lambda_w) = 0.590$ for $\lambda_w \geq 1.08$
resistance $V_{bw,Rd} = (\chi_w\cdot f_y\cdot h_w\cdot t_w) / (3^{1/2}\cdot\gamma_{M1}) = 4945.45$ kN
design value of resistance $V_{b,Rd} = V_{bw,Rd} = 4945.4$ kN
load capacities $V_{b,Rd,min} = 4379.9$ kN, $V_{bw,Rd,min} = 4379.9$ kN
verification: $V_{Ed}/V_{b,Rd,min} = 0.751 < 1$ **ok.**

interaction between shear, internal moment and axial force
utilization due to shear buckling of the web $\eta_3 = V_{Ed}/V_{bw,Rd,min} = 0.751 > 0.5$
utilization due to plate buckling $\eta_1 = M_{Ed}/M_{pl,Rd} = 0.663 > \eta_{1,lim} = M_{f,Rd}/M_{pl,Rd} = 0.587$
with $M_{pl,Rd} = 48493.1$ kNm, $M_{f,Rd} = 28454.4$ kNm
verification: $\eta_1 + (1-\eta_{1,lim})\cdot(2\cdot\eta_3-1)^2 = 0.767 < 1$ **ok.**

total utilization: $U = 0.931 < 1$ **ok.**

Final Result

maximum utilization [Lk 1]: max U = 0.970 < 1 ok.

verifications succeeded

Regulations

DIN EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;
Deutsche Fassung EN 1990:2002 + A1:2005 + A1:2005/AC:2010, Ausgabe Dezember 2010
DIN EN 1990/NA, Nationaler Anhang zur DIN EN 1990, Ausgabe Dezember 2010

DIN EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
Deutsche Fassung EN 1993-1-1:2005 + AC:2009, Ausgabe Dezember 2010
DIN EN 1993-1-1/NA, Nationaler Anhang zur DIN EN 1993-1-1, Ausgabe Dezember 2010

DIN EN 1993-1-5, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-5: Plattenförmige Bauteile;
Deutsche Fassung EN 1993-1-5:2006 + AC:2009, Ausgabe Dezember 2010
DIN EN 1993-1-5/NA, Nationaler Anhang zur DIN EN 1993-1-5, Ausgabe Dezember 2010